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January 28, 2016

JN 15538

Willows Preparatory School
12280 Redmond-Woodinville Road
Redmond, Washington 98052

Attention: Yuka Shimizu
via email: yukashimizu@bcacademy.com

Subject: **Transmittal Letter – Geotechnical Engineering Study and Geologically Hazardous Areas Report**
Proposed Willows Preparatory School Development
12348 Redmond-Woodinville Road Northeast
Redmond, Washington

Dear Ms. Shimizu:

We are pleased to present this geotechnical engineering report for the new building for the Willows Preparatory School facility in Redmond. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design considerations for foundations, retaining walls, slope stability, and temporary excavations. This work was authorized by your acceptance of our proposal, P-9329, dated December 10, 2015.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

Thor Christensen, P.E.
Senior Engineer

cc: **S+L Architects** – Shin Goto
via email: sgoto@sandlarchitects.com

TRC/MRM:at

GEOTECHNICAL ENGINEERING STUDY AND
GEOLOGICALLY HAZARDOUS AREAS REPORT
Proposed Willows Preparatory School Development
12348 Redmond-Woodinville Road Northeast
Redmond, Washington

We were provided with a site plan of existing conditions and floor plans prepared by S+L Architects dated December 2015. We were also provided with a site plan showing the planned development dated January 2016, also prepared by S+L Architects. Based on these plans, we understand that the development will consist of a new structure for the Willows Preparatory School, which currently includes a building (#12280) located on a separate parcel to the southeast of the site. For the purposes of this report, the site is considered to be the western half of Tax Lot # 26226059071, the parcel that contains the addresses 12320, 12328, and 12348 Redmond-Woodinville Road Northeast, which are the addresses of the existing structures according to the King County iMap website.

The development will consist of a new two-story building with a partial basement having a floor elevation of about 74 feet. The main floor will have an elevation close to 86 feet. An excavation about 10 feet deep will be necessary for the proposed basement, which will not extend below approximately the southern 45 feet of the southeastern "leg" of the building. Currently, we understand that the second floor will only be present in the western part of the building. Walkways around the structure will have an approximate elevation of 82 feet. The southeastern section of the building will be very close to the toe of an existing terraced slope and the base of a stacked masonry retaining wall a few feet tall. The northeast corner of the building will be close to the top of a short sideslope of an existing stormwater detention pond. An open plaza to the south of the southeastern "leg" of the building will connect it to the existing Willows Preparatory School to the southeast by extending across a sloping area to an existing driveway. The plaza will be up to 15 feet above the existing ground surface in the sloping area. Existing pavement areas north, west, and south of the new building may be repaved, but no significant changes in surface elevation are expected.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

<u>SITE CONDITIONS</u>

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site in Redmond. The site is bordered to the west by Redmond-Woodinville Road Northeast, the south by the Washington Cathedral church building (#12300), the southeast by the existing Willows Preparatory School (#12280), and the north and east by undeveloped land.

The site is currently developed with four one-story buildings that are surrounded with asphalt and concrete pavement. King County Assessor records show that they were constructed between 1956 and 1971. Those buildings cover most of the proposed building footprint. A stormwater detention pond is northeast of the existing buildings. The base of the pond is approximately 6 feet below the elevation of the existing on-site pavement, which extends up to a stacked concrete ultra block wall that serves as a vehicle barrier. The wall is two blocks high and extends one block (2.5 feet) above the pavement. South and east of the pond, the ground slopes upward to the driveway and parking located north of the Willows Preparatory School. This sloping ground has an inclination of 35 percent.

An asphalt driveway wraps around the north and east sides of the pond, extending from the site up to the parking lot adjacent to the Willows Preparatory School. East of that driveway the ground surface slopes down toward the northeast with an inclination of about 25 percent. There are existing off-site homes accessed from Northeast 124th Street at the base of this sloping ground.

Along the northern portion of the western property line is a newer concrete retaining wall that appears to retain a cut that was necessary for reconstruction and/or widening of Redmond-Woodinville Road Northeast. This wall has a maximum height of approximately 9 feet at the northwest corner of the site. The wall continues toward the north, becoming taller on the adjacent property.

Immediately southeast of the southeastern on-site building (#12328), the ground slopes steeply up to a curved driveway adjacent to the existing Willows Preparatory School. This slope has a height of about 15 feet. At the steepest portion of the slope most of the elevation change is achieved with three tiered block retaining walls. The lowest wall is very close to the existing building. It appears that at least the top of the slope, as well as the material behind the walls, is composed of fill. Based on a December 5, 2005 letter report prepared by GeoEngineers, this slope was steep before the additional grading and terraced walls completed as a part of the 2007 construction of the Willows Preparatory School and the associated driveway that extends along the top of this slope. Before the more recent permitted grading, the slope had been oversteepened by excavation for the original construction of building #12328. We observed no indications of recent instability in this slope, and none were noted in the 2005 GeoEngineers report.

From our review of the Redmond Critical Area maps, the southeastern slope has been mapped as a Landslide Hazard Area. This steep slope, as well as the moderately-sloped ground to the east, is also mapped as an Erosion Hazard Area. The Critical Areas in the development area, and within 50 feet of its boundary, are indicated on Plate 2. These areas were confirmed by visual observation during several visits to the property.

SUBSURFACE

The subsurface conditions were explored by drilling five test borings at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The borings were drilled on January 13, 2016 using a trailer-mounted, hollow-stem auger drill. Samples were taken at approximate 2.5- and 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test borings, and obtained representative samples of the soil encountered. The Test Boring Logs are attached as Plates 3 through 7.

Soil Conditions

Test Borings 1 and 4 were located in or near the southeastern leg of the proposed building. Both encountered silt that was dense at depths of 2.5 feet and extended to about 17 to 22 feet. This soil has been glacially compressed. The silt was underlain by very dense silty sand to the base of Test Boring 1 at a depth of 21.4 feet, and by very dense sand with gravel and then silt to the base of Test Boring 4 at a depth of 31 feet.

It was not possible to drill Boring 4 closer to the top of the steep slope off the southeast corner of the proposed building, due to the presence of an extensive amount of utilities located in the existing driveway. Based on our observations, we expect that there is both newer fill and pre-existing fill comprising the upper portion of this steep slope. The base of the slope appears to have been cut into the dense, native soil when building #12328 was originally built.

Test Borings 2 and 3 were completed near the north side of the new building footprint. They revealed loose soil to a depth of about 5 feet, consisting of fill in Test Boring 2 and silt with organics in Boring 3. This silt found in Boring 3 may be the original topsoil layer. These loose soils were underlain by layers of medium-dense silt, sand, and silty sand with gravel to a depth of 10 feet. Below the medium-dense soils in Test Boring 2 the boring found very dense sandy gravel with silt to about 17 feet, and then very dense silty sand to the base of the boring at 21.4 feet. In Test Boring 3 the medium-dense materials were followed by about 5 feet of dense silt and then very dense silty sand with gravel to the base of the exploration 21.5 feet below the surface.

Test Boring 5 was located close to the southwest corner of the proposed building. It encountered loose fill and wood to a depth of 4 feet. From that depth to approximately 12 feet, stiff to very stiff clayey silt was revealed, and was underlain by medium-dense sand to silty sand to about 17 feet. That material was followed by very dense silty sand with gravel that continued to the bottom of the boring at 21.5 feet.

Groundwater Conditions

Perched groundwater seepage was observed only in Boring 2, at a depth of 15 feet. The test borings were completed following several months of wet weather, but left open for only a short time period. Therefore, the seepage level on the log represents the location of transient water seepage and may not indicate the static groundwater level. Groundwater levels encountered during drilling can be deceptive, because seepage into the boring can be blocked or slowed by the auger itself. It should be noted that groundwater levels vary seasonally with rainfall and other factors.

Based on the conditions encountered in the borings, we do not expect that extensive groundwater will be encountered within the native soils above the expected excavation level. However, perched water within the looser soils and old fill could be found, particularly following extended wet weather.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. If a transition in soil type occurred between samples in the borings, the depth of the transition was interpreted. The relative densities and moisture descriptions indicated on the test pit boring logs are interpretive descriptions based on the conditions observed during drilling.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The test borings conducted for this study encountered medium-dense to dense or stiff soils at the proposed foundation elevations. The composition of the materials encountered in our explorations was fairly variable, but predominately consisted of silt. Conventional foundations bearing directly on these competent native soils will provide suitable support to the proposed building. The silt soils anticipated to be present over the majority of the building subgrade can be disturbed very easily, especially when wet. To reduce the potential for disturbance we recommend that the subgrade be covered with several inches of crushed rock as soon as the footings are excavated. This is especially important in wet conditions.

Given that the site has undergone previous grading, it is possible that fill or disturbed soils could be encountered. If it is necessary to overexcavate below the planned footing grades to reach suitable bearing soils, either the foundations can be extended downward, or the overexcavation can be backfilled using compacted rock fill.

One geotechnical concern for the project is the portion of the proposed basement excavation that will be close to the adjacent southeastern slope. Temporary excavations taller than 4 feet can have an inclination as steep as 1:1 (H:V), measured from the top and bottom of the entire excavation and any adjacent slope. However, in this area an adequately-sloped temporary cut would require the removal of the access drive that crosses the top of the steep slope. It is unlikely that this will be acceptable. As a result, we anticipate that excavation shoring will be needed to maintain the stability of the existing slope when the excavation for the foundations is made. Recommendations for soldier pile shoring are presented in the ***Temporary Shoring*** section. Less aggressive shoring methods, such as stacked ecology blocks, are not appropriate for the soil and topographic conditions that exist in the southeastern corner of the site. The shoring will have to consider the likelihood that the lower existing block wall will have to be removed for the new building excavation. This will probably require that the soldier pile shoring be installed upslope of the lowest block wall.

Shoring should also be provided for any temporary excavations that extend below a 1.5:1 (H:V) zone sloping down from the footings of existing structures, such as the church building to the south.

The plaza extending between the southeastern leg of the building and the existing paved driveway will be located over a steep slope area that has been modified by past cutting, filling, and block wall construction. Even though this steep slope was created by previous legal grading, it would still be classified as a Landslide Hazard Area under the Redmond Zoning Code. The recommendations of this report are intended to prevent the planned development from adversely impacting the stability of this slope during construction, and to prevent the new construction from being damaged by any future instability. In its current condition, the slope does not exhibit any indications of instability. However, in order to maintain appropriate long-term stability under both static and seismic conditions, we recommend that the foundation walls of the new building be tall enough to backfill this area to a final grade of no steeper than 40 percent (2.5H:1V). The compacted backfill placed between the new foundation wall and the steep slope must consist of imported granular soil compacted in general accordance with our recommendations. After completion of the backfilling, the completed slope should be provided with appropriate erosion protection either by planting it, or by covering it with the proposed plaza that will extend south from the new building. Once the backfilling and erosion control have been completed, no buffer or setback from this Landslide Hazard Area is required. Also, no post-construction mitigation or monitoring of the stabilized Landslide Hazard Area will be needed.

The plaza constructed to the south of the building will be at least partially underlain by old fill placed for the previous grading of the steep slope. If that old fill is not removed, it may result in some long-term ground settlement beneath the on-grade plaza, which could translate to more-than-typical cracking of concrete, and potentially some downsets in the paved surface. If this is undesirable, the plaza could be constructed as a reinforced structure supported with pipe piles. Recommendations for piles are presented later in the ***Pipe Piles*** section.

The steep slope in the southeast corner of the site is also mapped as an Erosion Hazard Area. With the exception of the shoring installation at the toe of the slope, the remainder of the slope should be left undisturbed until it is to be backfilled, as recommended above. Any areas of bare soil on the steep slope should be covered with plastic sheeting in wet weather. Once the slope is backfilled and permanent erosion protection is installed, no further mitigation or monitoring for this Erosion Hazard Area will be needed. For the general site construction away from the steep slope, we anticipate that a wire-backed silt fence will be needed around the downslope sides of any cleared areas. Existing pavements, ground cover, and landscaping should be left in place wherever possible to minimize the amount of exposed soil. Any large areas of bare soil created outside of the work area will need to be covered with plastic, mulch, gravel, wood chips, straw or another appropriate material. This will be important not only to reduce the potential for silty runoff in wet conditions, but also to minimize dust in dry weather. Rocked staging areas and construction access roads should be provided to reduce the amount of soil or mud carried off the property by trucks and equipment. Wherever possible, the access roads should follow the alignment of planned pavements. Trucks should not be allowed to drive off of the rock-covered areas. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Any silty water generated by the project must be prevented from flowing off site. This may require the use of temporary holding tanks (a.k.a. "Baker" tanks). Following clearing or rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface. On most construction projects, it is necessary to periodically maintain or modify temporary erosion control measures to address specific site and weather conditions.

An underslab drainage system is prudent for the basement portion of the new building. This is discussed below in ***Slabs-On-Grade***.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking and bathing. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The designer or architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a build up of excessive water vapor within the planned structure.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

We recommend including this report, in its entirety, in the project contract documents. This report should also be provided to any future property owners so they will be aware of our findings and recommendations.

SEISMIC CONSIDERATIONS

In accordance with the International Building Code (IBC), the site soil profile within 100 feet of the ground surface is best represented by Site Class Type D (Stiff Site Class). As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.26g and 0.48g, respectively.

The site soils that will support the building are not susceptible to seismic liquefaction because of their compact nature and the absence of near-surface groundwater. This statement regarding liquefaction includes the knowledge of the peak ground acceleration that is anticipated under a 1-in-2,500-year seismic event, which is the Maximum Considered Earthquake (MCE) as required by the IBC.

CONVENTIONAL FOUNDATIONS

The proposed structure can be supported on conventional continuous and spread footings bearing on undisturbed, medium-dense to dense, native soil. We recommend that continuous and individual spread footings have minimum widths of 16 and 24 inches, respectively. Exterior footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface for protection against frost and erosion. The local building codes should be reviewed to determine if different footing widths or embedment depths are required. Footing subgrades must be cleaned of loose or disturbed soil prior to pouring concrete. Depending upon site and equipment constraints, this may require removing the disturbed soil by hand.

An allowable bearing pressure of 4,000 pounds per square foot (psf) is appropriate for footings supported on competent native soil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil will be about one-inch, with differential settlements on the order of one-half-inch in a distance of 30 feet along a continuous footing with a uniform load.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level, well-compacted fill.

We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.40
Passive Earth Pressure	350 pcf

Where: pcf is Pounds per Cubic Foot, and Passive Earth Pressure is computed using the Equivalent Fluid Density.

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend maintaining a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate values.

PIPE PILES

Three-inch-diameter pipe piles driven with a 650- or 800- or 1,100-pound hydraulic jackhammer to the following final penetration rates may be assigned an allowable capacity of 6 tons.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (650-pound hammer)	FINAL DRIVING RATE (800-pound hammer)	FINAL DRIVING RATE (1,100-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
3 inches	12 sec/inch	10 sec/inch	6 sec/inch	6 tons

Note: The refusal criteria indicated in the above table are valid only for pipe piles that are installed using a hydraulic impact hammer carried on leads that allow the hammer to sit on the top of the pile during driving. If the piles are installed by alternative methods, such as a vibratory hammer or a hammer that is hard-mounted to the installation machine, numerous load tests to 200 percent of the design capacity would be necessary to substantiate the allowable pile load. The appropriate number of load tests would need to be determined at the time the contractor and installation method are chosen.

As a minimum, Schedule 40 pipe should be used. The site soils are not highly organic, and are not located near salt water. As a result, they do not have an elevated corrosion potential. Considering this, it is our opinion that standard "black" pipe can be used, and corrosion protection, such as galvanizing, is not necessary for the pipe piles.

Pile caps and grade beams should be used to transmit loads to the piles. Isolated pile caps should include a minimum of two piles to reduce the potential for eccentric loads being applied to the piles. Subsequent sections of pipe can be connected with slip or threaded couplers, or they can be welded together. If slip couplers are used, they should fit snugly into the pipe sections. This may require that shims be used or that beads of welding flux be applied to the outside of the coupler.

FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

PARAMETER	VALUE
Active Earth Pressure *	35 pcf (Level Backslope) 55 pcf (2.5H:1V backslope)
Passive Earth Pressure	350 pcf
Coefficient of Friction	0.40
Soil Unit Weight	130 pcf

Where: pcf is Pounds per Cubic Foot, and Active and Passive Earth Pressures are computed using the Equivalent Fluid Pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls beyond what we have presented in the table above, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired. The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 8**H** pounds per square foot (psf), where **H** is the design retention height of the wall. This applies to a level backslope condition. A surcharge of 12**H** psf should be used where the backfill is sloped at 2.5H:1V. Using these increased pressures, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. The on-site soils are very silty and should not be reused as wall backfill. The silty soils have poor drainage characteristics and a low compacted strength.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. Also, subsurface drainage systems are not intended to handle large volumes of water from surface runoff. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls to reduce the potential for surface water to percolate into the backfill. Water percolating through pervious surfaces (pavers, gravel, permeable pavement, etc.) must also be prevented from flowing toward walls or into the backfill zone. The compacted subgrade below pervious surfaces and any associated drainage layer should therefore be sloped away. Alternatively, a membrane and subsurface collection system could be provided below a pervious surface.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled ***General Earthwork and Structural Fill*** contains additional recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing, and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a build up of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact an experienced envelope consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

The ***General***, ***Slabs-On-Grade***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

SLABS-ON-GRADE

The building floors can be constructed as slabs-on-grade atop competent native soil, or on structural fill. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill.

Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. This can affect moisture-sensitive flooring, cause imperfections or damage to the slab, or simply allow excessive water vapor into the space above the slab. All interior slabs-on-grade should be underlain by a capillary break drainage layer consisting of a minimum 4-inch thickness of clean gravel or crushed rock that has a fines content (percent passing the No. 200 sieve) of less than 3 percent and a sand content (percent passing the No. 4 sieve) of no more than 10 percent. Pea gravel or crushed rock are typically used for this layer.

Considering that the basement level will be lower than the surrounding grade, and will be underlain by low permeability soil, it would be prudent to install underslab drainage in addition to the typical perimeter footing drains. This is intended to provide added protection for the basement slab area in the event that subsurface water bypasses one of the footing drains. Such an underslab system typically consists of a minimum 8-inch layer of drain rock or pea gravel with 4-inch perforated pipes buried in the gravel on 15- to 20-foot centers. These perforated drainage pipes would then be connected to the foundation drainage system. A typical underslab drainage detail is attached to the end of this report as Plate 9.

As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI also notes that vapor *retarders* such as 6-mil plastic sheeting have been used in the past, but are now recommending a minimum 10-mil thickness for better durability and long term performance. A vapor retarder is defined as a material with a permeance of less than 0.3 perms, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where vapor retarders are used under slabs, their edges should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection. If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.01 perms when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

TEMPORARY SHORING

Soldier pile walls would be constructed after making planned cut slopes, and prior to commencing the mass excavation, by setting steel H-beams in a drilled hole and grouting the space between the beam and the soil with concrete for the entire height of the drilled hole. The contractor should be prepared to case the holes or use the slurry method if caving soil is encountered. Excessive ground loss in the drilled holes must be avoided to reduce the potential for settlement on adjacent properties. If water is present in a hole at the time the soldier pile is poured, concrete must be tremied to the bottom of the hole.

As excavation proceeds downward, the space between the piles should be lagged with timber, and any voids behind the timbers should be filled with pea gravel, or a slurry comprised of sand and fly ash. Treated lagging is usually required for permanent walls, while untreated lagging can often be utilized for temporary shoring walls. Temporary vertical cuts will be necessary between the soldier piles for the lagging placement. The prompt and careful installation of lagging is important, particularly in loose or caving soil, to maintain the integrity of the excavation and provide safer working conditions. Additionally, care must be taken by the excavator to remove no more soil between the soldier piles than is necessary to install the lagging. Caving or overexcavation during lagging placement could result in loss of ground on neighboring properties. Timber lagging should be designed for an applied lateral pressure of 30 percent of the design wall pressure, if the pile spacing is less than three pile diameters. For larger pile spacings, the lagging should be designed for 50 percent of the design load.

If permanent building walls are to be constructed against the shoring walls, drainage should be provided by attaching a geotextile drainage composite with a solid plastic backing, similar to Miradrain 6000, to the entire face of the lagging, prior to placing waterproofing and pouring the foundation wall. These drainage composites should be hydraulically connected to the foundation drainage system through weep holes placed in the foundation walls.

Soldier Pile Wall Design

Temporary soldier pile shoring that is cantilevered and that has a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 30 pounds per cubic foot (pcf). This pressure should increase to 55 pcf for shoring located below the existing steep southeastern slope.

Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. Slopes above the shoring walls will exert additional surcharge pressures. These surcharge pressures will vary, depending on the configuration of the cut slope and shoring wall. We can provide recommendations regarding slope surcharge pressures when the preliminary shoring design is completed.

It is important that the shoring design provides sufficient working room to drill and install the soldier piles, without needing to make unsafe, excessively steep temporary cuts. Cut slopes should be planned to intersect the backside of the drilled holes, not the back of the lagging.

DRAINAGE CONSIDERATIONS

We anticipate that permanent foundation walls will be constructed against the shoring walls. Where this occurs, a plastic-backed drainage composite, such as Miradrain, Battledrain, or similar, should be placed against the entire surface of the shoring prior to pouring the foundation wall. Weep pipes located no more than 6 feet on-center should be connected to the drainage composite and poured into the foundation walls or the perimeter footing. A footing drain installed along the inside of the perimeter footing will be used to collect and carry the water discharged by the weep pipes to the storm system. Isolated zones of moisture or seepage can still reach the permanent wall where groundwater finds leaks or joints in the drainage composite. This is often an acceptable risk in unoccupied below-grade spaces, such as parking garages. However, formal waterproofing is typically necessary in areas where wet conditions at the face of the permanent wall will not be tolerable. If this is a concern, the permanent drainage and waterproofing system should be designed by a specialty consultant familiar with the expected subsurface conditions and proposed construction.

Footing drains placed inside the building or behind backfilled walls should consist of 4-inch, perforated PVC pipe surrounded by at least 6 inches of 1-inch-minus, washed rock wrapped in a non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the level of a crawl space or the bottom of a floor slab, and it should be sloped slightly for drainage. Plate 8 presents typical considerations for footing drains and Plate 10 presents a typical shoring drain. All roof and surface water drains must be kept separate from the foundation drain system.

If the structure includes an elevator, it may be necessary to provide special drainage or waterproofing measures for the elevator pit. If no seepage into the elevator pit is acceptable, it will be necessary to provide a footing drain and free-draining wall backfill, and the walls should be waterproofed. If the footing drain will be too low to connect to the storm drainage system, then it will likely be necessary to install a pumped sump to discharge the collected water. Alternatively, the elevator pit could be designed to be entirely waterproof; this would include designing the pit structure to resist hydrostatic uplift pressures.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing even a few inches of free draining gravel underneath the vapor retarder limits the potential for seepage to build up on top of the vapor retarder.

Groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to a building should slope away at least 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls. A discussion of grading and drainage related to pervious surfaces near walls and structures is contained in the ***Foundation and Retaining Walls*** section. Water from roof, storm water, and foundation drains should not be discharged onto slopes; it should be tightlined to a suitable outfall located away from any slopes.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. It is important that existing foundations be removed before site development. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, behind permanent retaining or foundation walls, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

Fills placed on sloping ground should be keyed into the medium-dense native soils. This is typically accomplished by placing and compacting the structural fill on level benches that are cut into the competent soils. The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction. The following table presents recommended relative compactions for structural fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

Structural fill that will be placed in wet weather should consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the test borings are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in test borings. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of Bellevue Children's Academy / Willows Preparatory School and its representatives for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with our understanding of current local standards of practice, and within the scope of our services. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

During the construction phase, we will provide geotechnical observation and testing services when requested by you or your representatives. Please be aware that we can only document site work we actually observe. It is still the responsibility of your contractor or on-site construction team to verify that our recommendations are being followed, whether we are present at the site or not.

The scope of our work did not include an environmental assessment, but we can provide this service, if requested.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 7	Test Boring Logs
Plate 8	Typical Footing Drain Detail

Plate 9	Typical Underslab Drainage Detail
Plate 10	Typical Shoring Drain Detail

We appreciate the opportunity to be of service on this project. Please contact us if you have any questions, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

Thor Christensen, P.E.
Senior Engineer

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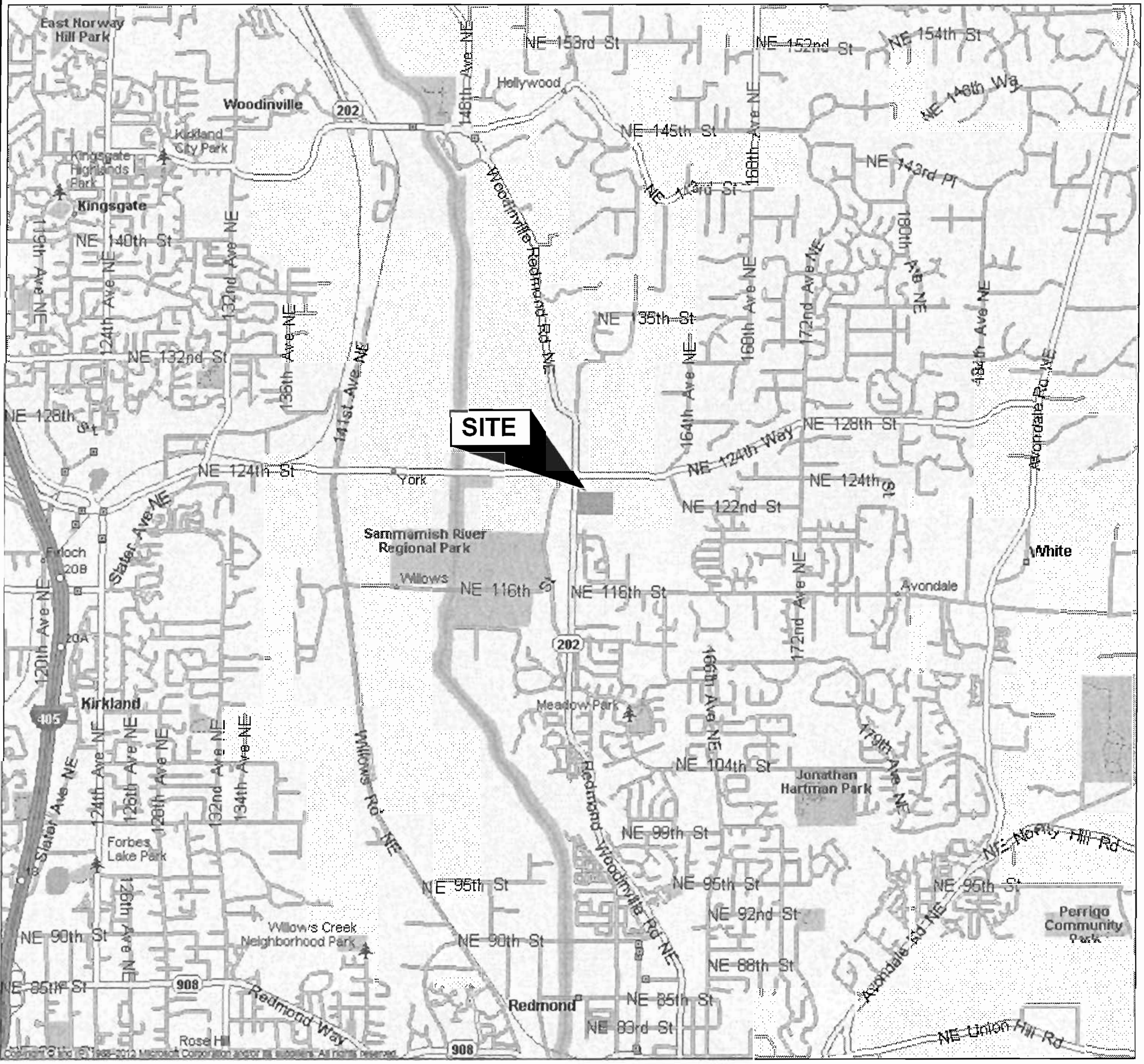
STATE OF WASHINGTON
27845
REGISTERED
PROFESSIONAL ENGINEER

1/28/16

Marc R. McGinnis, P.E.
Principal

TRC/MRM:at

NORTH



(Source: Microsoft MapPoint, 2013)

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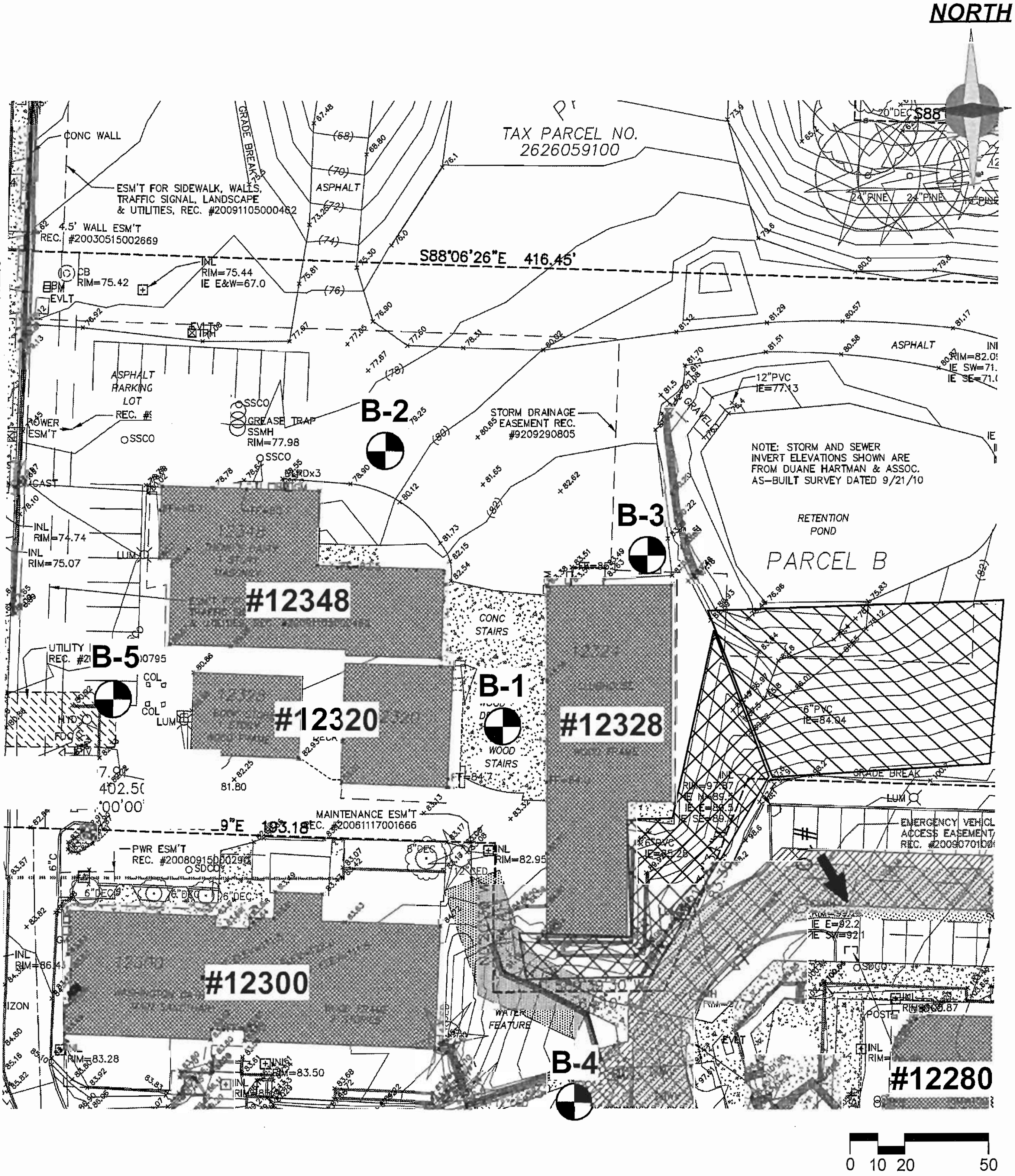
VICINITY MAP

12348 Redmond - Woodinville Road N.E.
Redmond, Washington

Job No:	Date:		Plate:
15538	Jan. 2016		1



Legend:

- Boring Location
- Landslide and Erosion Hazard Area
- Erosion Hazard Area Only

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SITE EXPLORATION PLAN

12348 Redmond - Woodinville Road N.E.
Redmond, Washington

Job No:	Date:		Plate:
15538	Jan. 2016		2

BORING 1

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
0–4.5			32	1	ML	4 inches of concrete over; Gray-brown sandy SILT, fine-grained, non-plastic, moist, loose to medium-dense -becomes dense
4.5–18			23	2	ML	Gray SILT, non-plastic to plastic, moist, medium-dense to very stiff
			33	3		-becomes dense and non-plastic
			38	4		
18–21.4			90/ 11"	5	SM	Gray silty SAND, fine to coarse-grained, moist, very dense

*** Test boring was terminated at 21.4 feet on January 12, 2016.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG

12348 Redmond - Woodinville Road N.E.
Redmond, Washington

Job	Date:	Logged by:	Plate:
15538	Jan. 2016	TRC	3

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	3 inches of asphalt over 6 inches of rock spalls over; Gray to brown silty SAND with gravel, fine to coarse-grained, moist, loose **(FILL)**
			7	1		
5			13	2	ML	Gray-brown SILT, non-plastic, moist, medium-dense
					SW SM	Gray SAND with silt, fine to coarse-grained, wet, medium-dense
			25	3	ML	Brown SILT with sand and gravel in some zones, fine to coarse-grained, non-plastic, medium-dense
10			72	4	GW	Gray-brown sandy GRAVEL with silt, fine to coarse-grained, moist, very dense
15		▼	50/6"	5		-reduced silt content
20			50/5"	6	SM	Gray silty SAND, fine to medium-grained, moist, very dense
25						
30						

* **Test boring was terminated at 21.4 feet on January 12, 2016.**
* **Groundwater was encountered at 15 feet during drilling.**

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TEST BORING LOG

12348 Redmond - Woodinville Road N.E.
Redmond, Washington

Job	Date:	Logged by:	Plate:
15538	Jan. 2016	TRC	4

BORING 3

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
						2 inches of asphalt over;
			4	1	OL	Brown to gray-brown organic SILT, non-plastic, moist, very loose
					ML	Gray clayey SILT, plastic, moist, medium-stiff
5			13	2	ML / SM	Rust-brown mottled gray-brown SILT to silty SAND with gravel, fine to coarse-grained, non-plastic to plastic, moist, medium-dense
			20	3	SM / SW	Gray-brown silty SAND with gravel to SAND with gravel, fine to coarse-grained, moist to wet, medium-dense
10			32	4	ML	Rust-brown mottled gray-brown SILT, non-plastic, moist, dense
15			60	5	SM	Rust-brown mottled gray-brown silty SAND with gravel, fine to coarse-grained, moist, very dense
20			66	6		-reduced silt content, becomes gray
25						
30						

*** Test boring was terminated at 21.5 feet on January 12, 2016.**
*** No groundwater was encountered during drilling.**

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TEST BORING LOG

12348 Redmond - Woodinville Road N.E.
Redmond, Washington

Job	*Date:*	*Logged by:*	*Plate:*
15538	Jan. 2016	TRC	5

BORING 4

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
0						2-1/2 inches of asphalt over;
					ML	Rust-brown mottled gray-brown SILT, non-plastic, moist, dense
			40	1		
5			29	2		-becomes gray
10			35	3		-becomes plastic in some zones
15			36	4		
20			59	5		-becomes very dense and non-plastic
25			85/ 11"	6	SW	Gray SAND with gravel, fine to coarse-grained, wet, very dense
30			75/ 12"	7	ML	Gray SILT, non-plastic, moist, very dense

* **Test boring was terminated at 31 feet on January 12, 2016.**
* **No groundwater was encountered during drilling.**

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TEST BORING LOG

12348 Redmond - Woodinville Road N.E.
Redmond, Washington

Job	Date:	Logged by:	Plate:
15538	Jan. 2016	TRC	6

BORING 5

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	2-1/2 inches of asphalt over; Brown silty SAND, fine to coarse-grained, wet, loose **(FILL)** -wood, possible stump
5			18	1	ML	Brown clayey SILT, plastic, moist, very stiff
			13	2		-becomes stiff and light brown
10			17	3		-with a 2" layer of fine to coarse-grained sand
15			14	4	SP/SM	Rust-brown mottled gray-brown SAND to silty SAND, fine to medium-grained, moist, medium-dense
20			75	5	SM	Rust-brown mottled gray-brown silty SAND with gravel, fine to coarse-grained, moist, very dense
25						
30						

* **Test boring was terminated at 21.5 feet on January 12, 2016.**
* **No groundwater was encountered during drilling.**

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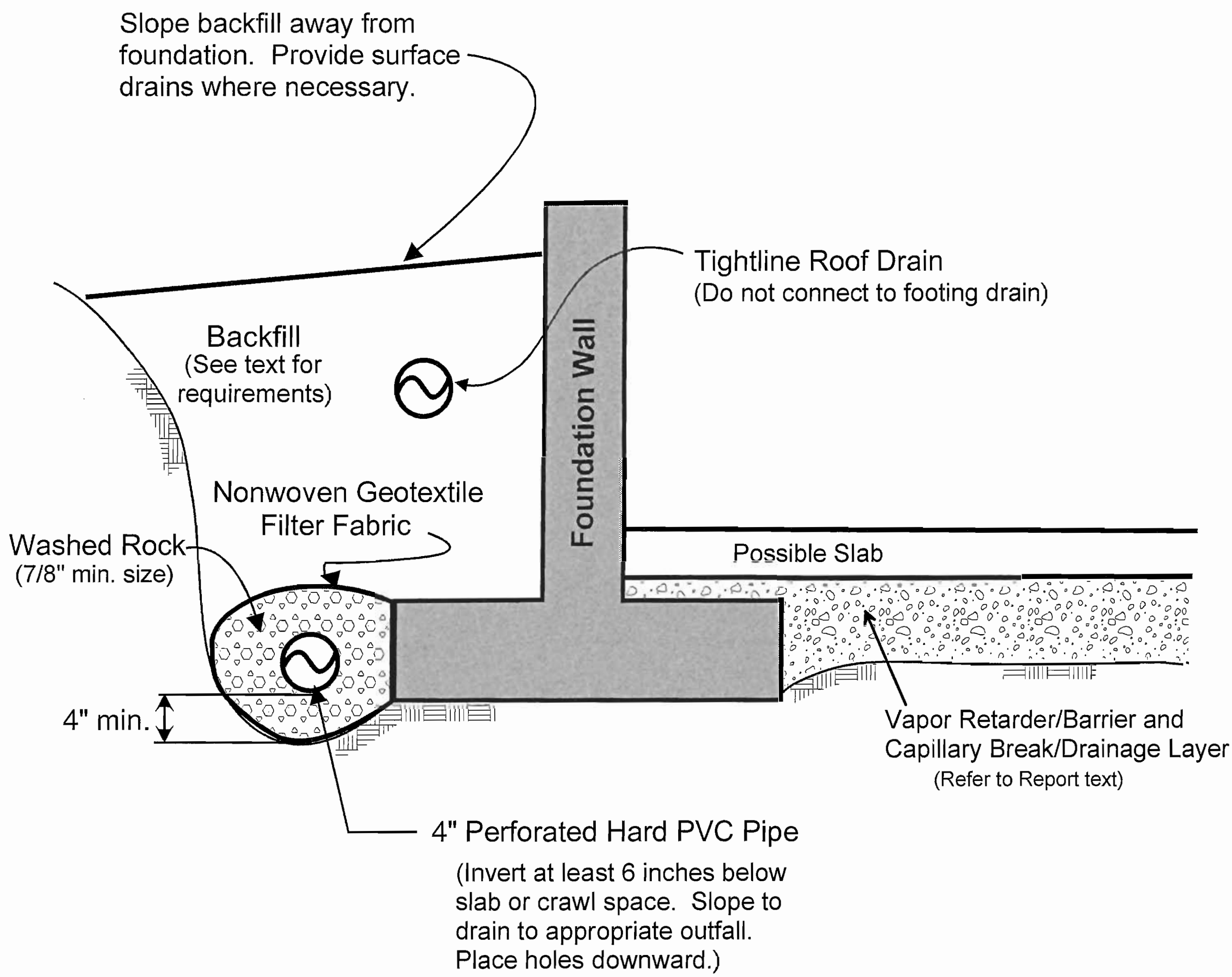
TEST BORING LOG
12348 Redmond - Woodinville Road N.E.
Redmond, Washington

Job	Date:	Logged by:	Plate:
15538	Jan. 2016	TRC	7



NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

(2) Refer to report text for additional drainage, waterproofing, and slab considerations.

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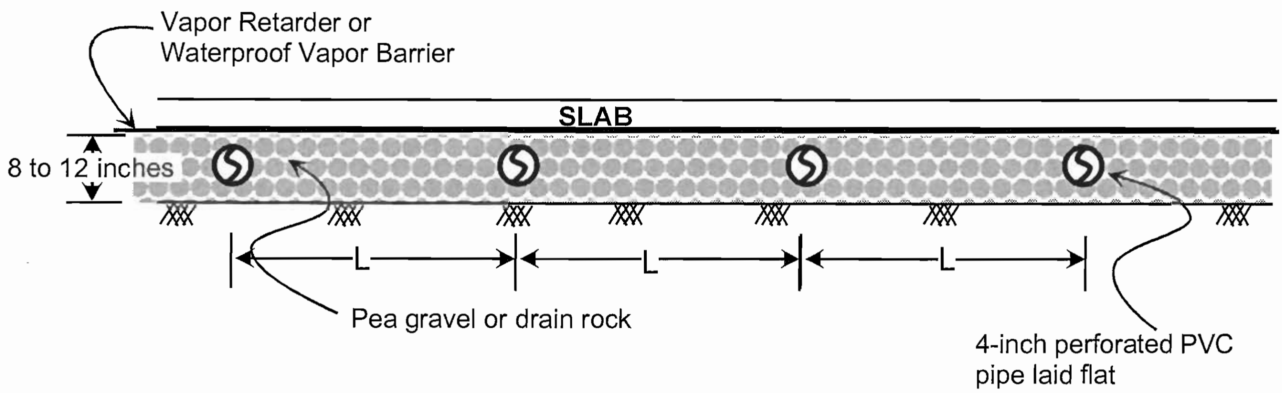
FOOTING DRAIN DETAIL

12348 Redmond - Woodinville Road N.E.
Redmond, Washington

Job No:	Date:		Plate:
15538	Jan. 2016		8



NOTES:

(1) Refer to the report text for additional drainage and waterproofing considerations.
(2) The typical maximum underslab drain separation (L) is 15 to 20 feet.
(3) No filter fabric is necessary beneath the pipes.
(4) The underslab drains and foundation drains should discharge to a suitable outfall.

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TYPICAL UNDERSLAB DRAINAGE

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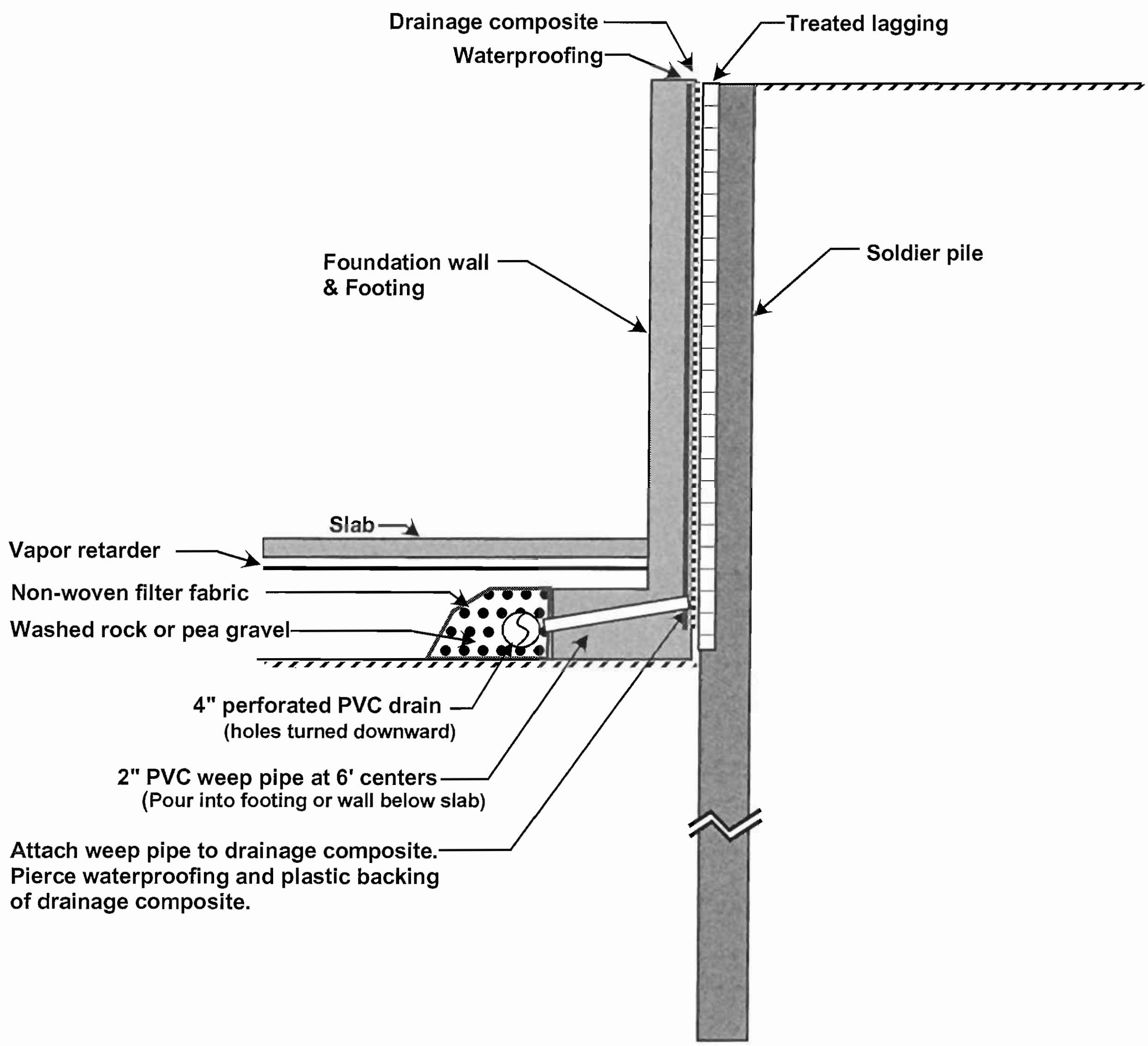
Redmond, Washington

Job No:	Date:		Plate:
15538	Jan. 2016		9



<u>Note</u> - Refer to the report for additional considerations related to drainage and waterproofing.

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SHORING DRAIN DETAIL

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Redmond, Washington

Job No:	Date:		Plate:
15538	Jan. 2016		10